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STABILITY ANALYSIS OF A TAILINGS DAM: EXISTING STATE AND PLANNED HEIGHTENING

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ABSTRACT

This paper presents the studies carried out to determine the stability of a tailings dam for its existing state and for a planned heightening. The dam is made of rockfill; it is about 3 500 m long and its maximum height is presently 38 m. The tailings are heterogeneous; they originate from three industrial enterprises in the region and are transported mixed with water, through supply pipelines. The facility is built on foundation made up of clays, underlain by marl. The present study consists of seepage analysis; consolidation analysis to assess the stress states at different stages during the building of the dam and the filling of the pond; time-domain earthquake analysis to evaluate the maximum response of the structure during seismic excitation; and slope stability analysis. The analyses are performed by means of the finite-element method through the GEO-SLOPE program. Based on the analyses, it is concluded that the structural safety of the existing facility is sufficient for both usual and unusual (earthquake) load combinations. Several alternatives are considered for possible heightening of the tailings dam and the corresponding factors of safety are computed for each one. The best heightening option is selected based on technical and economical consideration.

INTRODUCTION

The Padina tailings dam is located in northeast Bulgaria, near the city of Varna. It creates storage to deposit the tailings from three industrial enterprises in the region: the Solvay-Sodi factory for production of soda ash, the Agropolichim factory for production of mineral fertilizers, and the Deven thermal power plant. It is a rockfill dam, about 3 500 m long, and its maximum height is presently 38 m. The facility is built on foundation made up of clays, underlain by marl. The total volume of the tailings deposited so far is 35 million cubic meters. The dam is built and, respectively, the created pond is filled on consecutive stages. The following ones have been completed until now: Stage I from El. 9 to El. 23, Stage II to El. 32, Stage III to El. 38, Stage IV to El. 42, and Stage V to El. 47. The present study concerns the stability of the existing facility, i.e. at the end of Stage V, as well as the feasibility of upgrading the dam. Several alternatives for heightening are investigated and the optimal one is selected based on technical and economical considerations.

The numerical analyses are performed on 2-D FEM models of three characteristic cross sections of the dam-tailings-foundation system, which are denoted as Profile 19, Profile 22 and Profile 24. The computations are carried out by the GEO-SLOPE package of geotechnical software programs.

The study of the stability of the existing state consists of: (1) seepage analyses to determine the phreatic surface in the investigated dam-tailings-foundation system, in assumption of steady-state seepage; (2) consolidation analyses to assess the stress states at different stages during the building of the dam and the filling of the pond; (3) earthquake analyses to evaluate the maximum response during seismic excitation; and (4) slope stability analyses for usual and unusual (earthquake) load combinations. The latter analyses are carried out by the FEM and checked by the Morgenstern-Price method.

The structural safety of the dam for each of the investigated upgrading alternatives is assessed for the cross-section with minimum factors of safety at its present state. Detailed seepage and slope stability analyses are carried out for usual load combination for each of the considered stages of upgrading. Additionally, consolidation and seismic slope stability calculations are conducted for the proposed heightening alternative.

In the following are presented the load combinations considered in the study and the corresponding design material characteristics, the main results from the above analyses, and the conclusions for the stability of the tailings dam at its present state. Further are given details on the considered upgrading alternatives and the technical and economical analysis carried out to select the optimal upgrading.

LOADS AND LOAD COMBINATIONS

The following loads and load combinations have been considered in the present study:

- Usual load combination: .... $\mathbf{G_w+U}$
- Unusual load combination: .... $\mathbf{G_w+U+E}$

where $\mathbf{G_w}$ denotes self weight, **U** stands for pore-water pressure, and **E** means the inertia loads caused by seismic excitation due to Design Earthquake.

The seismic loading is specified according to the Bulgarian Seismic Code (1987). The earthquake analyses are performed by means of a direct time-step integration procedure. To this end, horizontal acceleration time history is applied along the boundaries of the FEM models of the investigated dam cross sections. The acceleration time history is generated by the SIMQKE program in a manner that its design spectrum envelopes the one specified in the Code. The peak ground acceleration, corresponding to the Design Earthquake (1 000-year return period) is taken according to the Seismic Map of Bulgaria as PGA=0.10 g.

DESIGN MATERIAL CHARACTERISTICS

The design material characteristics have been defined based on thorough field and laboratory investigations carried out recently on the Padina tailings dam and foundation materials by Energoproekt-Hydropower (2005) and FUGRO. The assumption pre-determining the selection of the design material characteristics is that the dam-tailings-foundation system responds to the static loads in drained conditions, whereas in the case of strong earthquake excitation, excess pore-water pressure is generated in the cohesive materials whose response would be in un–drained conditions. The assumption for the presence of drained conditions during the usual load combination is confirmed by the results from the consolidation analyses carried out which show that no excess pore-water pressure is generated during the filling of the pond.

According to the above assumptions, drained–condition effective–stress shear strengths in terms of angle of internal friction and cohesion (ϕ' and c') are specified for all the materials in the computations of the usual load combination. On the other hand, in the earthquake analyses (unusual load combination), un-drained–condition shear strengths (s_u) are specified for the cohesive materials, i.e. for the tailings and the foundation clays. Figure 1 shows the material regions considered for Profile 19, Profile 22 and Profile 24 for static and seismic analysis. The general material properties and the drained–condition effective–stress shear strengths are given in Table 1; the un-drained–condition shear strengths are given in Table 2.

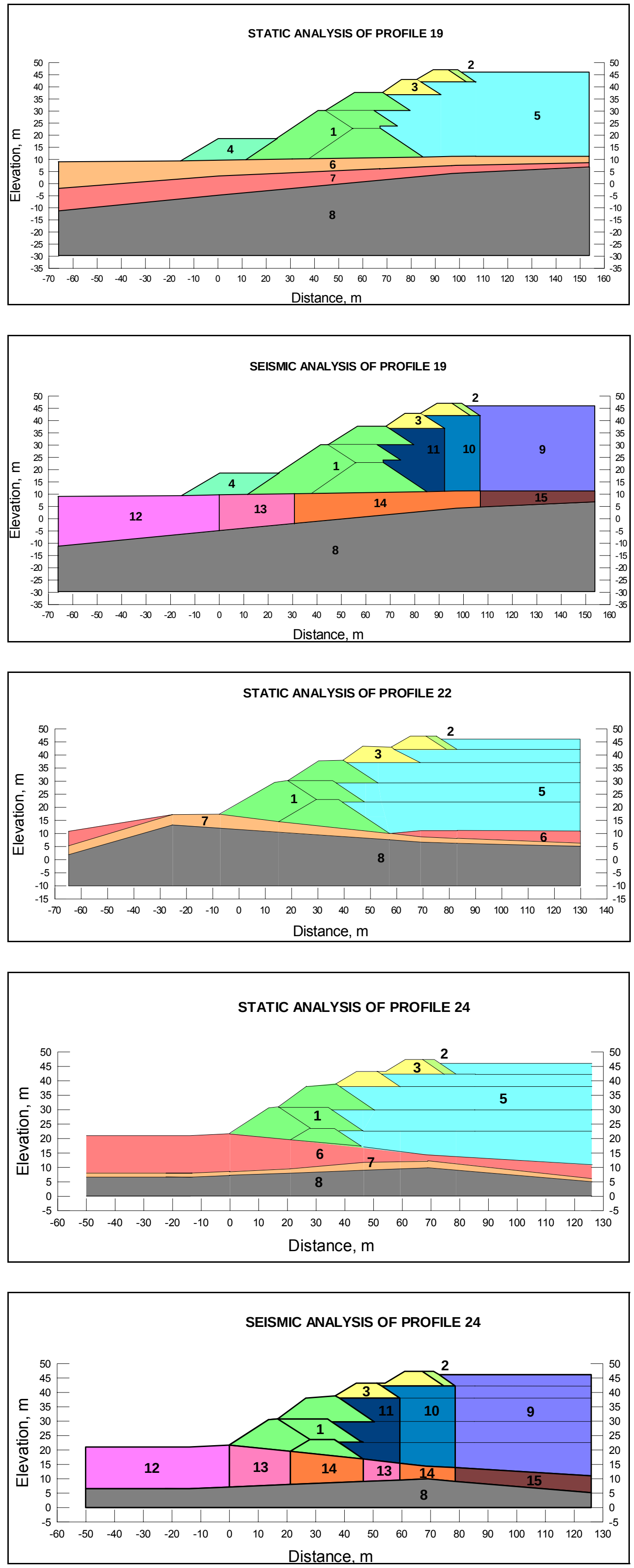

Fig. 1. Material regions considered in usual and unusual load combinations for Profiles 19, 22, and 24

Table 1. General material properties and drained–condition effective–stress shear strengths

No	Material	Hydraulic conductivity	Mass density	Density of solid constituents	Void ratio	Deformation modulus	Poisson's ratio	Modulus of elasticity	Drained shear strength	
		k	ρ_n	ρ_s	e	E_0	ν	E	ϕ'	c'
		m/s	g/cm^3	g/cm^3	-	kPa	-	kPa	°	kPa
1	Rockfill – Lyulyaka Quarry	$1*10^{-5}$	1.86			50 000	0.30	75 000	38.00	20.00
2	Fill	$5*10^{-6}$	1.89			45 000	0.31	70 000	33.70	22.50
3	Rockfill – G. Sakar Quarry	$1*10^{-5}$	1.86			50 000	0.30	75 000	38.00	20.00
4	Counterfill	$5*10^{-6}$	1.89			45 000	0.31	70 000	33.70	22.50
5	Tailings	$5*10^{-8}$	1.33	0.58	4.19	4 000	0.35	16 100	34.80	0.00
6	Alluvial Clay	$1*10^{-7}$	1.98	1.58	0.74	12 900	0.34	16 300	24.65	0.00
7	Marly Clay	$1*10^{-9}$	2.22	1.92	0.45	20 000	0.33	38 000	19.50	0.00
8	Marl	$1*10^{-8}$	2.40	1.98	0.30	50 000	0.30	75 000	24.50	30.00

Table 2. Un-drained – condition shear strengths

No	Material	Strength Model	Cohesion at Top, c	Rate of Increase	Maximal Cohesion	Un-drained shear strength
			c	-	c_{max}	s_u
			kPa	-	kPa	kPa
9	Tailings in the Pond	s_u=f(depth)	5	3	-	$s_u=5+3d$
10	Tailings beneath the Dike of Stage 5	s_u=f(elevation)	30	3	-	$s_u=30+3z_1$
11	Tailings beneath the Dike of Stage 4	s_u=f(elevation)	30	3	-	$s_u=30+3z_2$
12	Clay Foundation at the Dam Toe	s_u=f(depth)	25	6.25	125	$s_u=25+6.25d\leq125$
13	Clay Foundation of the Counterfill	s_u=f(depth)	50	6.25	125	$s_u=50+6.25d\leq125$
14	Clay Foundation of the Dam	Undrained (ϕ=0)	-	-	125	$s_u=125$
15	Clay Foundation of the Pond	Undrained (ϕ=0)	-	-	100	$s_u=100$

SEEPAGE ANALYSES

The seepage analyses are carried out in order to determine the total head and the pressure fields in the facility at the end of Stage V (analysis of existing state); as well as at the end of the respective upgrading stages. An assumption for steady-state seepage conditions is made. The results from the seepage analyses are used in the slope stability computations carried out by means of a limit-equilibrium method (Morgenstern-Price). The total head contours for Profile 19 are given in Fig. 2.

To calibrate and check the seepage models, additional computations are performed in which the boundary conditions are specified according to records from the piezometers installed in the tailings dam. The computed phreatic lines from the calibration analyses compare well with the piezometric records.

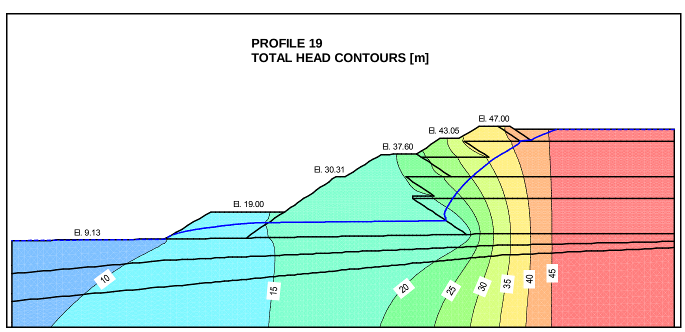

Fig. 2. Total head contours for Profile 19, end of Stage V.

CONSOLIDATION ANALYSES

GEO-SLOPE modules SIGMA/W and SEEP/W are run simultaneously to perform a fully-coupled consolidation analysis. SIGMA/W calculates the deformations resulting from pore-water pressure changes while SEEP/W calculates transient pore-water pressure changes.

The purpose of the consolidation analyses is to determine the stress and the pore-water pressure fields at different stages of building of the dam and filling of the pond. The computed stress is then used as initial condition for the FEM slope stability analyses for usual load combination.

The sequence of the dam construction and tailings pond filling is modeled in a manner to simulate as close as practical the actual works and processes. Figure 3 shows the modeled stages of building and impounding. Linear-elastic material behavior is assumed.

The results from the consolidation computations show that any excess pore-water pressure in the tailings and the foundation clay completely dissipates at the end of Stage V (i.e. 31 years after the beginning). It is therefore concluded that the consolidation of the facility is completed and the static loads are withstood in consolidated and drained conditions. This justifies the selection of drained-condition shear strength parameters for the slope stability analyses.

The deformed mesh for the end of Stage V is given in Fig. 4 and the horizontal and vertical effective stresses are presented in Fig. 5 and Fig. 6, respectively. It is important to note the negative (i.e. tensile) effective horizontal stresses at the top of the dikes of Stages III and IV. These negative stresses evidence of the possibility of formation of cracks at the said locations. Such cracks have indeed been observed on site and subsequently properly treated.

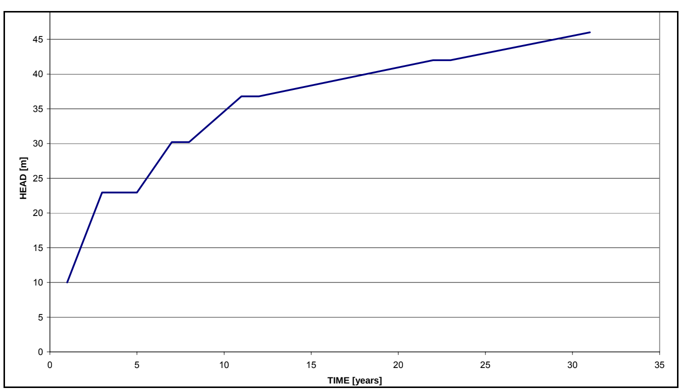

Fig. 3. Rising of the total head in the tailings lake.

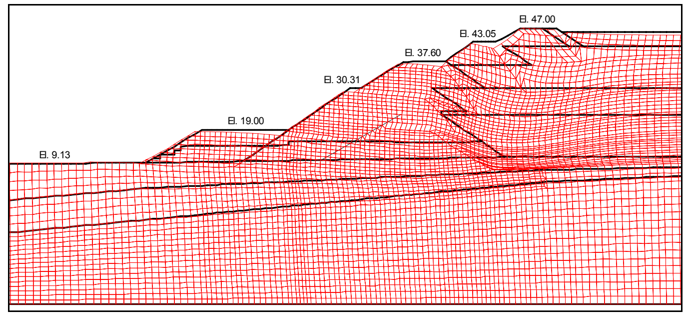

Fig. 4. Deformed mesh for Profile 19, end of Stage V.

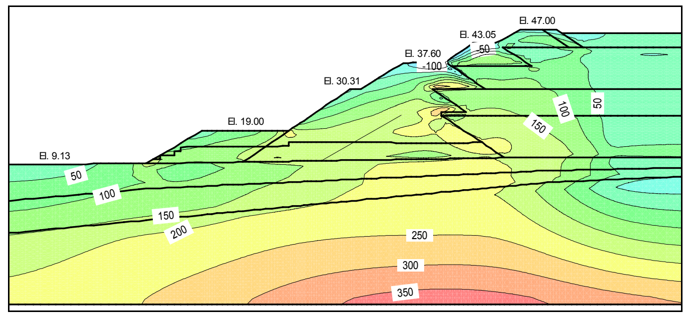

Fig. 5. Effective horizontal stresses for Profile 19, end of Stage V.

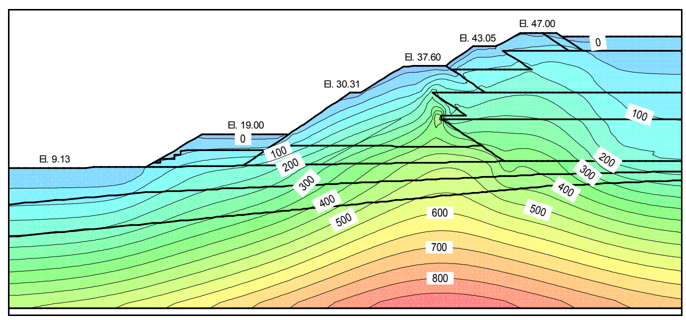

Fig. 6. Effective vertical stresses for Profile 19, end of Stage V.

STATIC SLOPE STABILITY ANALYSES

The computations for the existing state are performed by the FEM with initial conditions the results for the strains, stresses and pore-water pressures obtained from the consolidation analysis. For comparison, in addition are performed computations by the Morgenstern-Price limit equilibrium method with initial condition the pore-water pressure field obtained through seepage analysis as described in the previous section. The two sets of slope stability analysis give approximately the same values of the minimum factor of safety (FS). For Profile 19, the computed FS for the end of Stage V are 1.41 (FEM) and 1.39 (Morgenstern-Price). The FS for Profile 22 are 2.05 by both methods. For Profile 24, FS=1.52 (FEM) and FS=1.53 (Morgenstern-Price). Figures 7 and 8 show the critical slip surfaces and the corresponding factors of safety for Profile 19 from both kinds of analysis.

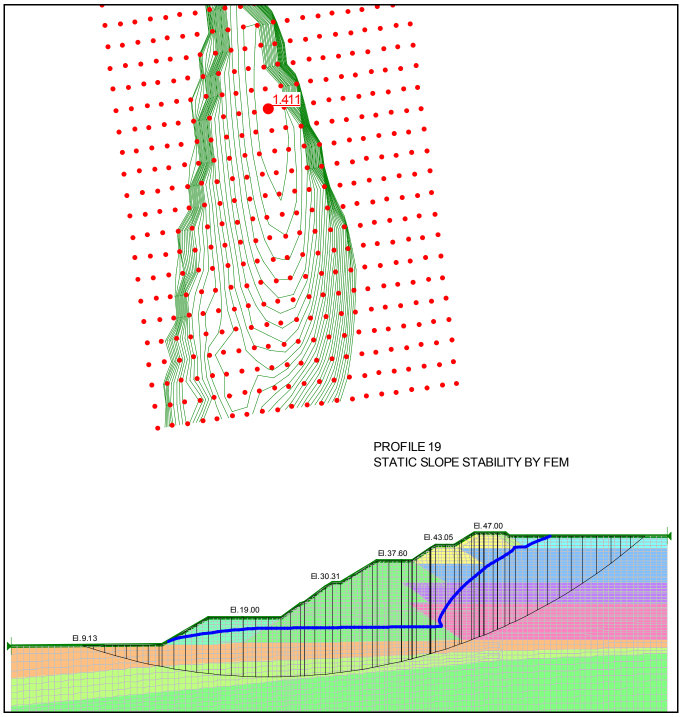

Fig. 7. Static slope stability of Profile 19 computed by FEM. Critical slip surface and minimum FS.

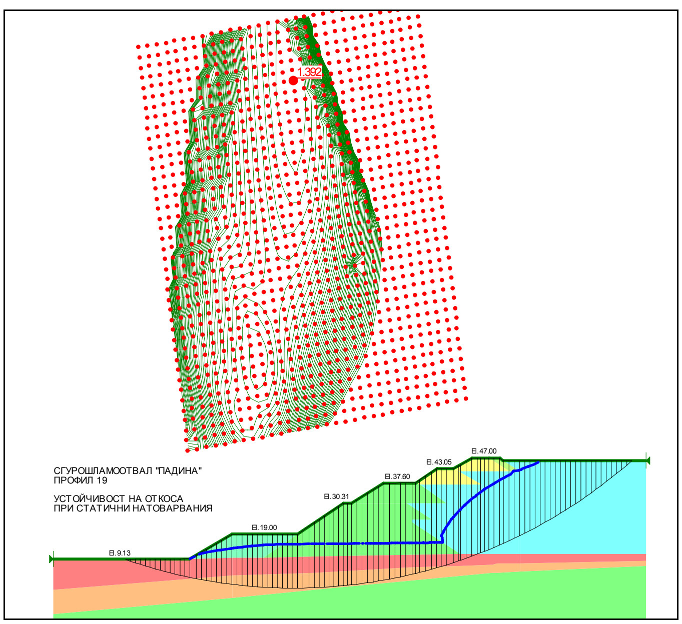

Fig. 8. Static slope stability of Profile 19 computed by the Morgenstern-Price Method. Critical slip surface and min FS.

EARTHQUAKE SLOPE STABILITY ANALYSES

The response of the structure to a 30-sec design earthquake excitation is obtained by a time-step integration analysis. The computations are performed by the QUAKE/W module of GEO-SLOPE with initial conditions the results from the consolidation analysis. The behavior of the rockfill and the marl is assumed linear-elastic; equivalent-linear soil properties are assigned to the tailings and the clay materials. The analysis is performed at a time step of 0.02 sec and the structural response is saved for each step. The instance of the most critical stress state is determined based on the displacement time-history of a node at the top of the tailings dam. The maximum horizontal displacement (in downstream direction) is computed at t=15.06 for Profile 19 and t=5.68 sec for Profile 24. The displacement time-histories computed for the investigated nodes of Profile 19 and 24 are given in Fig. 9 and Fig. 10, respectively.

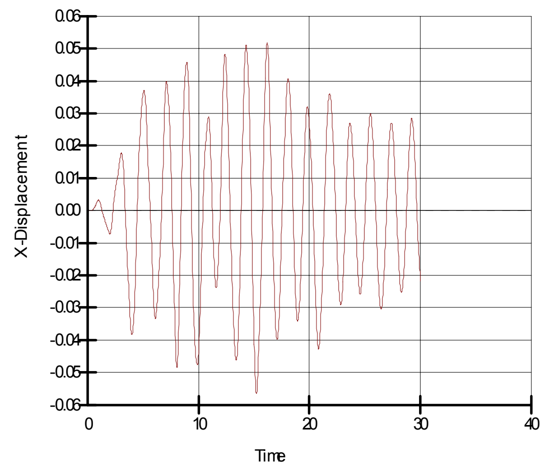

Fig. 9. Displacement time history of a point at the center of the tailings dam crest at El.47 of Profile 19.

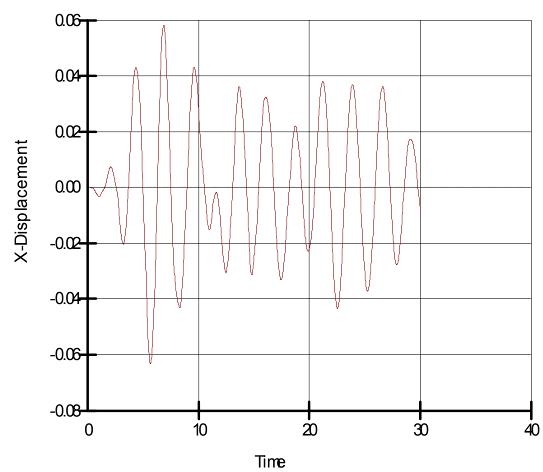

Fig. 10. Displacement time history of a point at the center of the tailings dam crest at El.47 of Profile 24.

Similar to the static slope stability analyses, the dynamic slope stability is computed by the finite-element method (for the instance of the assumed most critical state) and checked in a pseudo-dynamic analysis by the Morgenstern-Price limit equilibrium method. The obtained factors of safety for Profiles 19 and 24 are, respectively, FS=1.08 and FS=1.20.

CONCLUSIONS FROM THE ANALYSIS OF THE EXISTING STATE

Thorough static and earthquake analyses have been carried out on characteristic profiles of Padina tailings dam. The input geotechnical parameters data for these analyses have been supplied by extensive field and laboratory investigations. The computed phreatic surface, as well as vertical and horizontal displacements compare well with the data recorded through the monitoring system of the facility.

The computed factors of safety for the slope stability of the investigated characteristic cross sections of the tailings dam are given in Table 3 below.

Table 3. Summary of the computed factors of safety

	Usual load combination	Unusual load combination
Profile 19	1.40	1.08
Profile 22	2.05	-
Profile 24	1.50	1.20

Based on the static and earthquake analyses results, it is concluded that the tailings dam is safe for both usual and unusual load combinations, as the lowest Factors of Safety are obtained for Profile 19.

Nevertheless, in order to increase the stability of the part of the dam with type cross section corresponding to Profile 19, it has been decided to build additional rockfill on top of existing counterfill as shown in Fig. 11 below:

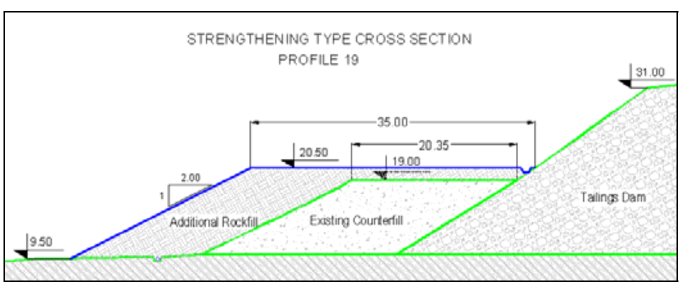

Fig. 11. Strengthening of the existing tailings dam. Profile 19 type cross section.

With the additional rockfill, the slope stability of Profile 19 increases to FS=1.66 for Usual Load Combination and FS=1.19 for Unusual Load Combination.

INVESTIGATED UPGRADING ALTERNATIVES

Prior to investigating the possible upgrading alternatives, the following limiting conditions have been defined:

- The release of clarified water from the tailings pond is through a reinforced concrete outlet pipeline of 1.7-m diameter. The load bearing capacity of the pipeline would become insufficient should the deposited waste exceed El.75.0.
- There is a minimum length of the beach necessary to provide clarification of the water released from the tailings pond. It is about 400 m.
- The available information on the deposited tailings and foundation materials from the geotechnical investigations carried out extends to about 60 m upstream of the Stage 5 dike.
- The existing infrastructure and instrumentation.

Taking into account the above limitations, five upgrading alternatives have been investigated, including the one proposed in the pre-feasibility study of 1991. The type cross sections for Profile 19 of the upgrading alternatives (denoted as ‘options’ in the drawings) are shown in Figures 12 to 16.

Detailed computations have been made to determine the added pond capacity and the bill of quantities for the upgrading stages and totally for each alternative. These data, along with the corresponding FS computed by the Morgenstern-Price method are given in Table 4.

It is noted that the slope stability computations have been made for a cross-section of Profile 19 without the proposed strengthening. It is also noted that each of Alternatives 3, 4 and 5 involves the installation of approximately 150 000 m^2 geogrid, 150 000 m^2 geotextile and 1 800 m belt drainage.

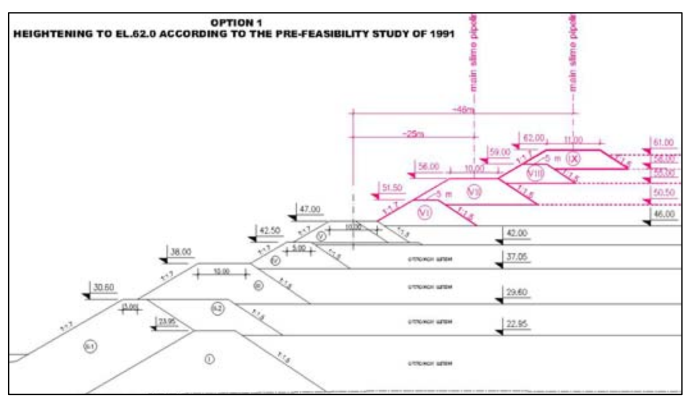

Fig. 12. Type cross section of upgrading Alternative 1.

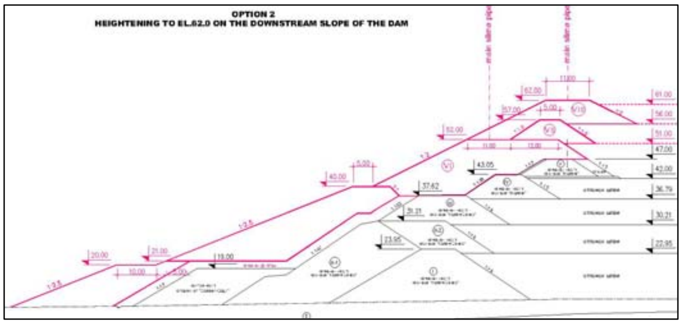

Fig. 13. Type cross section of upgrading Alternative 2.

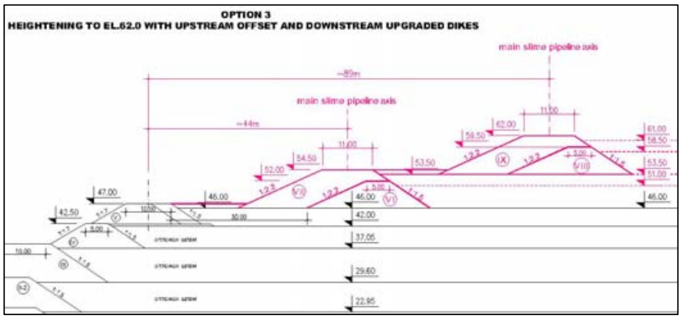

Fig. 14. Type cross section of upgrading Alternative 3.

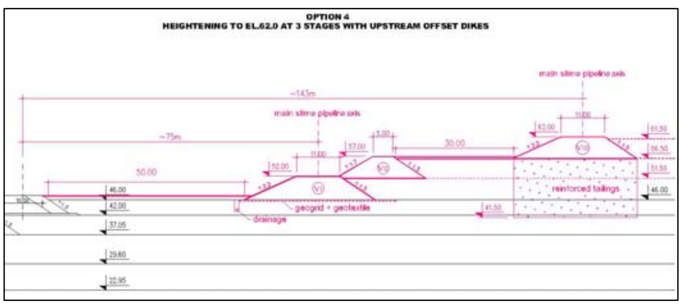

Fig. 15. Type cross section of upgrading Alternative 4.

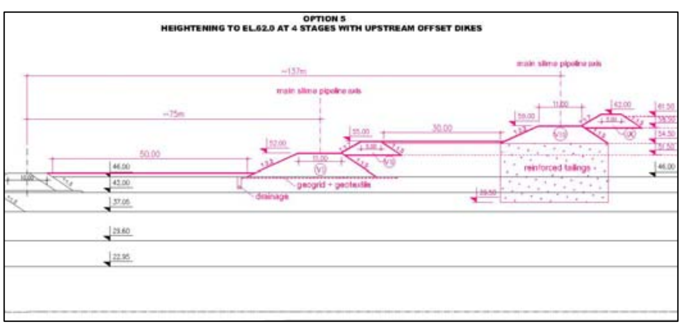

Fig. 16. Type cross section of upgrading Alternative 5.

Table 4. Factors of safety for global slope stability of the investigated upgrading alternatives (for Profile 19 without strengthening)

Alt. No.	Stage No.	El., m	FS Usual LC	Rockfill,m^3	Added capacity, mio m^3
1	VI	51.5	1.34	258 700	8.0
	VII	56	1.28	341 400	8.0
	VIII	59	1.23	154 300	5.5
	IX	62	1.17	220 900	5.0
			Total	975 300	26.5
2	VI	52	1.44	1 983 100	9.0
	VII	57	1.44	235 300	8.5
	VIII	62	1.40	540 600	9.0
			Total	2 759 000	26.5
3	VI	52	1.44	428 220	8.5
	VII	54.5	1.36	583 960	4.1
	VIII	59.5	1.45	418 860	7.8
	IX	62	1.38	574 340	3.6
		Total		2 005 380	24.0
4	VI	52	1.45	784 500	8.1
	VII	57	1.46	322 270	6.9
	VIII	62	1.47	476 580	6.3
			Total	1 538 350	21.3
5	VI	52	1.45	784 500	8.1
	VII	55	1.46	155 870	4.1
	VIII	59	1.47	354 770	5.0
	IX	62	1.45	150 000	3.8
			Total	1 445 140	21.0

TECHNICAL AND ECONOMICAL COMPARATIVE ANALYSIS OF THE UPGRADING ALTERNATIVES

The technical aspects related to the upgrading alternatives (options) are as follows:

Alternative 1: The heightening, as planned in the pre-feasibility study, cannot be implemented because the safety of the tailings dam for the all the stages of heightening is not sufficient.

Alternative 2: Heightening on the downstream slope of the dam up to El.62 is possible, the global stability of the tailings dam is ensured, but the operation of the following major existing works and installations would be affected: the sodabrine pipelines, the main distributing junction, the Devnya-Sindel state road, the drain channels at the toes of the dam, the installed instrumentation system for monitoring the behavior of the facility, and the electrical installations.

Alternative 3: If the upgrading dikes at El.52 (Stage VI) and 59.5 (Stage VIII) are offset upstream by 50 m from the respective previous stage, the required global seismic safety of the dam cannot be provided. However, the safety can be ensured by an offset of 70 m. The implementation of Stage VIII and Stage IX requires an offset that locates the dikes of these stages inside the lake. The lack of information regarding the properties of the deposited material that would form the foundation of these stages is a source of significant uncertainty. In any case, the foundation conditions at these locations will be poor. Therefore, significant and costly reinforcing measures would be required for the implementation of Stages VIII and IX.

Alternative 4: The stress and strain state analysis carried out indicates that local reinforcement of the foundation of the Stage VI dike (El.52) is necessary. In the proposed option, this reinforcement is planned by a combination of geogrid, geotextile and belt drainage. The heightening to El.62 (Stage VIII) requires much more significant and costly reinforcement for the Stage VIII dike foundation, as already explained for Alternative 3.

Alternative 5: This alternative is conceptually similar to Alternative 4. Local reinforcement of the foundation of Stage VI (El.52) dike is necessary and much more significant and costly reinforcement of the Stage VIII (El.59) dike is to be provided.

Based on detailed cost estimates of the considered alternatives and the significance of the involved additional measures necessary for their implementation, it has been proposed to carry out the upgrading up to El.57 according to Alternative 4. Subsequent heightening above El.57 could also be realized; to this end, however, it will be necessary to perform additional geotechnical investigations on the properties of the deposited wastes forming the foundation of the Stage VIII dike, as well as on the possibilities for its reinforcement.

Additional static analyses have been carried out to determine the local slope stability and settlement of the Stages VI and VII dikes of the proposed Alternative 4. The obtained FS=2.26 is sufficiently high to ensure the local slope stability. The settlement of the foundation of the dikes is approximately 1.30 m and the maximum tensile force in the geogrid is 15 kN/m' for SS30 geogrid. The global slope stability for unusual load combination (in case of earthquake of level Design Earthquake) has also been investigated for all stages of Alternative 4. The minimum FS is 1.13 and is considered sufficiently high.

The plan view of the tailings pond with the proposed upgrading is given in Fig. 17.

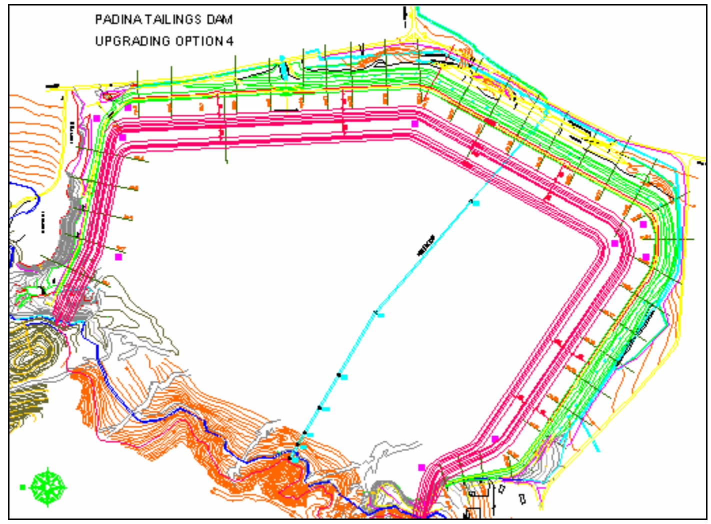

Fig. 17. Alternative 4 upgrading. Plan view.

CONCLUSIONS

This paper presents the studies carried out to determine the stability of a 38 m high, 3.5 km long tailings dam for its existing state and for a planned upgrading. Based on the obtained results, it is concluded that the structural safety of the existing facility is sufficient for both usual and unusual load combinations. It is, however, proposed to build an additional counterfill in order to increase the stability of the existing structure and also in view of the planned upgrading. The technical and economical aspects of several upgrading alternatives are considered in detail. It is concluded that the optimal upgrading can be achieved by offsetting the corresponding dikes upstream of the existing dam, i.e. inward the tailings pond. It is proposed to perform the upgrading at two stages, each of 5 m in height. Further upgrading by another 5 m is also possible; to this end, however, additional analyses based on data from detailed geotechnical investigations on the properties of the dike foundation are to be carried out.

ACKNOWLEDGEMENTS

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